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DISCRETE MODEL FOR SSSI BETWEEN CRITICAL STRUCTURES UNDER STRONG GROUND MOTION

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ABSTRACT

Usually, buildings in urban areas are designed by considering the response of structures in isolation i.e. a single structure, with no neighbouring structures. However, the existence of a high density of buildings in large cities inevitably results in the possibility of seismic interaction of adjacent buildings through the underlying soil that can produce an increase or decrease in seismic risk. Critical and important closely spaced structures, such as found in Nuclear power plants, are distinctly vulnerable to dynamic interaction, which should mandate full nonlinear SSSI analyses. In this study, we evaluated the effects of nonlinear Structure-Soil-Structure Interaction (SSSI) between two different buildings. A two-dimensional simple discrete nonlinear model is proposed that is described by a set of nonlinear differential equations of motion. The soil profile directly underneath foundation is modelled as a nonlinear phenomenological Bouc-Wen model and rotational interaction spring between buildings are assumed. We use an EC8 spectrum matched ground motion record. The Reweighted Volterra Series Algorithm (RVSA) is employed for the matching process. The results showed that there are unfavourable configurations of the two buildings that can produce important differences between nonlinear SSSI and nonlinear SSI. In the same way, it is demonstrated that the adverse effects of SSSI can be more pronounced when the nonlinear is considered.

Keywords: Structure-soil-structure interaction; nonlinear time-history analyses; Seismic analysis

1. INTRODUCTION

The importance of including the adverse structural effects of the structure-soil-structure interaction has received attention in the last decades, Luco and Contere [1], Korobi et al. [2] Wong and Trifunac [5], Lysmer et al [6] among others. Kitada et al. [8], Yano et al. [9], Hans et al. [10], Li et al. [11] and Aldaikh et al. [12] are experimental in situ studies. Numerical studies based on finite element method (FEM), boundary elements method (BEM) or a combination of these two FEM/BEM procedures with Bard et al. [13], Yahyai et al. [14], Padron et al. [15], Bolisetti and Whittaker [16], Alexander et al. [17], Aldaikh et al. [18], Chouw and Schmid [19] and Ogut and Fukuwa [20]. These studies have highlighted the importance of considering the dynamic coupling between several structures, especially in critical structures (such as nuclear reactor structures, hospitals and towers), where it is necessary to assess the seismic risk for the entire system and not only for an individual structure. The key factors that may control the seismic behavior are: (i) the inter-building distance, (ii) the direction of the alignment between foundations, (iii) the relative height and dynamic characteristics of adjacent buildings, (iv) the aspect ratio between height to width of buildings and (v) the general soil class.

Trombetta et al. [21-23] and Mason et al. [24] developed nonlinear experimental tests of specific building/foundation configurations. These studies represent important validation points for numerical models. However, these experiments are technically demanding, represent statistically a small sample and provide a limited parametric exploration of the problem. Some researcher's advocate using advanced computational models (FEA). Ghandil et al. [25] evaluate the SSSI, considering elasto-plastic frame hinges in the structure. Bolisetti and Whittaker [26] study the SSSI in a nonlinear model developed

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in the time-domain code LS-DYNA. Nevertheless, modelling a whole set of building configurations is very laborious. Thus, a large-scale parametric exploration of this problem requires a different approach. The alternative is to use parametric models, with a relatively limited number of degrees of freedom. These low-order models capture the most significant dynamic behaviour, have a relatively small number of system parameters and are computationally simple for exploring a huge number of generic cases.

Therefore, in this paper, we extend our previous parametric study on the SSSI of two linear buildings [17] to the case of nonlinear soil behaviour using the phenomenological Bouc-Wen model. We explore over 20000 different nonlinear systems. This computationally challenging study required the High-Performance Computing (HPC) machine, BlueCrystal, at the University of Bristol. The aim of this paper is to answer to the following two questions: (i) does the introduction of soil nonlinearity rise the adverse SSSI effects at a level that will increase the seismic risk in the structure and is not safe to neglect? and (ii) Is there evidence to suggest significant differences between nonlinear SSSI (the coupled building case) and nonlinear SSI (the uncoupled building case) analyses?

2. REDUCED ORDER MODEL FOR SSSI

2.1 Non-dimensional equation of motion

Consider the following system shown in Figure 1. It is a pair of building coupled by a rotational spring. Each building is described in terms of four degrees of freedom namely x_j to the translational DOFS and θ_j to the rotational DOFS, with $j \in [1,2]$. A known ground displacement field x_g is applied at both foundations. The kinetic energy T_E and potential energy U_E for this system are given by the following equations:

$$T_E = \frac{1}{2} \sum_{j=1}^2 \left(m_{bj} (\dot{x}_j + \dot{x}_g - h_j \dot{\theta}_j)^2 + m_{sj} r_j^2 \dot{\theta}_j^2 \right), \quad U_E = \sum_{j=1}^2 \left(\frac{1}{2} k_{bj} x_j^2 + \int M_j d\theta_j \right) + \frac{1}{2} \{ \kappa (\theta_2 - \theta_1)^2 \} \quad (1)$$

where h_j are the heights of buildings, m_{bj} are building masses, m_{sj} are the foundation/soil masses underneath building 1 and 2, r_j are the soil/foundation mass's radii of gyration, $m_{sj} r_j^2$ are the foundation/soil mass moments of inertia. k_{bj} are the linear building lateral stiffnesses, κ is the stiffness of inter-building soil rotational spring and b_j are the foundations' width. $M_j(\theta_j(t), y_j(t))$ are the nonlinear moments at the support springs, $y_j(t)$ are internal hysteretic rotation (history dependent of rotations θ_j) at time t , that controls the nonlinear response of the soil. The dimensional equation contains too many parameters, hence we seek parameter reduction through of removing all dimensional term. Thereby, we can introduce the following non-dimensional parameter groups,

$$\eta_j = \frac{h_j}{r_j}, \quad \alpha_j = \frac{m_{bj}}{m_{sj}}, \quad \lambda = \frac{m_{b2} r_2^2}{m_{b1} r_1^2}, \quad \omega_1^2 = \frac{k_{b1}}{m_{b1}}, \quad \omega_2^2 = \frac{k_{s1}}{m_{s1} r_1^2}, \quad \omega_3^2 = \frac{k_{b2}}{m_{b2}}, \quad \omega_4^2 = \frac{k_{s2}}{m_{s2} r_2^2} \quad (2)$$

$$\beta = \frac{r_1}{r_2}, \quad \varpi = \frac{\kappa}{m_{b1} r_1^2}, \quad \Omega_2 = \frac{\omega_2}{\omega_1}, \quad \Omega_3 = \frac{\omega_3}{\omega_1}, \quad \Omega_4 = \frac{\omega_4}{\omega_1}, \quad \Omega_0 = \frac{\varpi}{\omega_1} \quad (3)$$

To state our non-dimensional equation of motion, we introduce the change of variables $x_j = r_j u_j$, $x_g = r_1 u_g$ and $\tau = \omega_1 t$. ω_1 is the modal circular frequency on a fixed base (i.e. with no foundation/soil rotation) of the building 1, u_j are non-dimensional relative displacement of buildings to ground and u_g is the non-dimensional horizontal ground displacement (absolute). The Euler-Lagrange equations of motion can be stated thus,

$$\mathbf{M}\ddot{\mathbf{u}} + \mathbf{C}\dot{\mathbf{u}} + \mathbf{K}\mathbf{u} + \mathbf{q}(\theta, y) = \mathbf{f}\ddot{u}_g \quad (4)$$

where Newtonian dots above indicated derivatives with respect to scaled time τ , i.e. $(\dot{\bullet}) = \partial \bullet / \partial \tau$ and

$$(\ddot{\bullet}) = \partial^2 \bullet / \partial \tau^2.$$

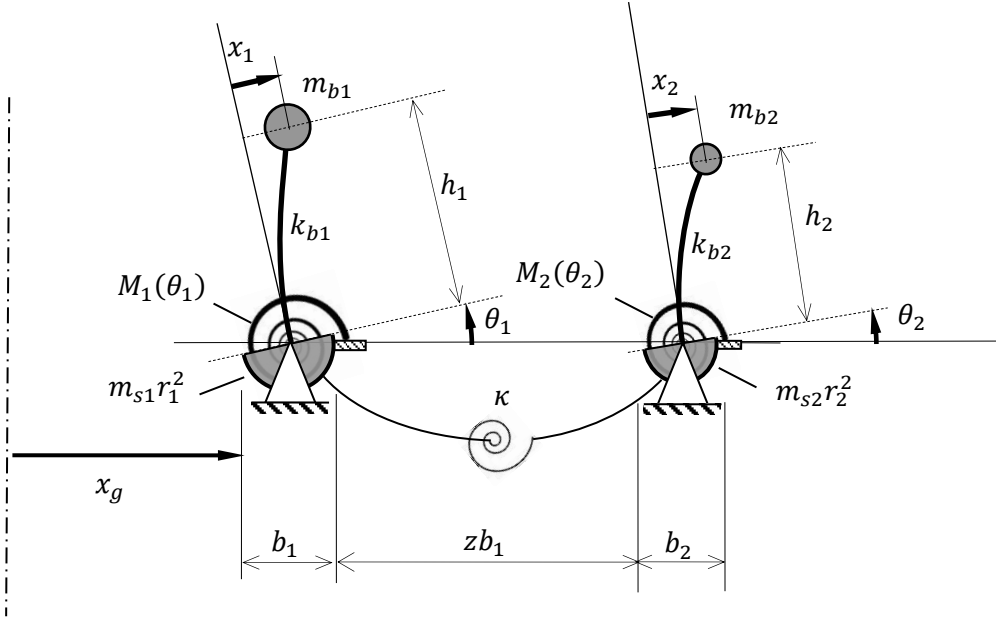


Figure 1. Two building system

The system's linear viscous damping matrix \mathbf{C} assume that each natural mode $n \in [1,4]$ is damped at $\xi_n = 0.05$ of critical damping. The nonlinearity in the equation (4) is contained in the vector $\mathbf{q}(\theta, y)$, where y_j is the internal degrees of freedoms that controls the nonlinear response of soil.

Equation (4) is expressed in terms of ten linear system parameters $\eta_1, \eta_2, \alpha_1, \alpha_2, \lambda, \Omega_0, \Omega_2, \Omega_3, \Omega_4$ and ω_1 plus eight constants that define the Bouc-Wen model. To further reduce the number of parameters, we make the following assumptions:

- (i) the same soil profile exists under both buildings, this means $k_{s1} = k_{s2}$
- (ii) both buildings have a similar square plan area of b^2 , where $r_1 = r_2 = 0.33b$
- (iii) both buildings have the same average density, ρ_b
- (iv) the buildings can have different heights, h_j
- (v) the buildings are spaced at some arbitrary distance from each other, zb_1 .

The dynamic mass of soil beneath buildings is equal to $m_s = 0.35b^3\rho_s$ and the mass of the buildings is $m_{bj} = \rho_b h_j b^2$, according to Newmark and Rosenblueth [27], where ρ_s and ρ_b are the densities of soil and building respectively. Parameters $\eta_1, \eta_2, \alpha_1, \alpha_2$ are contracted into two geometric parameters *Height ratio* ε and *Aspect ratio* s , where the proportionality constant c_1 is defined in table 2.

$$\varepsilon = \frac{h_2}{h_1}, \quad s = \frac{h_1}{b}, \quad \eta_1 = 3s, \quad \eta_2 = 3\varepsilon s, \quad \alpha_1 = \frac{c_1}{s}, \quad \alpha_2 = \frac{c_1}{\varepsilon s}, \quad c_1 = 0.35 \frac{\rho_s}{\rho_b} \quad (5)$$

Table 1. Linear elastic stiffness parameters for soil classes

Soil Class (sand)	$\rho_s [kg/m^3]$	$\mu []$	$c_1 []$	$c_2 []$	$V_s [m/s]$
Dense	2000	0.35	1.17	503.5	325
Medium	1600	0.30	0.93	468	250
Loose	1300	0.30	0.76	468	156

V_s is shear wave velocity of the soil in [m/s]. \bar{V}_s is the normalised non-dimensional shear wave velocity (to a reference of 1000 m/s) and soil constant c_2 . The interaction spring κ is modelled using an inverse cube relationship between κ and k_2 [17]. Rotational stiffnesses k_s are obtained by using an empirical formula (deducted by Gorbunov-Possadov et al. [28]), G_s is the initial tangent shear modulus of the soil and μ is the Poisson's ratio of the soil.

$$\kappa = q_k(z)q_2(z)k_s, \quad q_k(z) = -\frac{0.25}{(1+z)^3}, \quad q_2(z) = 1 + \frac{0.5}{(1+z)^3}, \quad k_s = \frac{1}{2} \frac{G_s b^3}{1-\mu} \quad (6)$$

Thus, we can re-express system matrices in terms of 3 geometric non-dimensional and one soil class.

$$\mathbf{M} = \begin{bmatrix} 1 & -3s & 0 & 0 \\ -3s & c_1 s^{-1} + 9s^2 & 0 & 0 \\ 0 & 0 & \varepsilon & -3\varepsilon^2 s \\ 0 & 0 & -3\varepsilon^2 s & c_1 s^{-1} + 9\varepsilon^3 s^2 \end{bmatrix}, \quad \mathbf{K} = \begin{bmatrix} 1 & 0 & 0 & 0 \\ 0 & c_1 c_2 q_2 q_k s \bar{V}_s^2 & 0 & -c_1 c_2 q_2 q_k s \bar{V}_s^2 \\ 0 & 0 & \varepsilon^{-1} & 0 \\ 0 & -c_1 c_2 q_2 q_k s \bar{V}_s^2 & 0 & c_1 c_2 q_2 q_k s \bar{V}_s^2 \end{bmatrix} \quad (7)$$

The nonlinear vector $\mathbf{q}(\theta, y)$ in its nondimensional form can be evaluate as:

$$\mathbf{q}(\theta, y) = B_1 c_1 c_2 q_2 s \bar{V}_s^2 \begin{bmatrix} 0 & 0 & 0 & 0 \\ 0 & 1 & 0 & 0 \\ 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 1 \end{bmatrix} \begin{bmatrix} u_1 \\ \theta_1 \\ u_2 \\ \theta_2 \end{bmatrix} + (1 - B_1) c_1 c_2 q_2 s \bar{V}_s^2 \begin{bmatrix} 0 & 0 & 0 & 0 \\ 0 & 1 & 0 & 0 \\ 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 1 \end{bmatrix} \begin{bmatrix} u_1 \\ y_1 \\ u_2 \\ y_2 \end{bmatrix} \quad (8)$$

As a measure of change in the response, we use the following performance measures.

$$U_j = u_j - 3 \frac{h_j}{b} \theta_j, \quad A_j = \ddot{u}_j + \ddot{u}_g - 3 \frac{h_j}{b} \ddot{\theta}_j, \quad \chi_{jj} = 100 \frac{[E_s(q_j)]_{SSSI} - [E_s(q_j)]_{SSI}}{[E_s(q_j)]_{SSI}} \quad (9)$$

Where U_j and A_j are the relative (sway + rotational) displacement and total (sway + ground + rotational) accelerations of buildings “j” in non-dimensional form. Additionally, we use the percentage change χ_{jj} in mean squared (the total power) response caused by building interactions, when moving from uncoupled (SSI) to coupled cases (SSSI). The total power spectral density E_s (which is based on all data points) is defined using Parseval's theorem. By using the Fourier transform of $q_j(\tau)$ we can obtain the power spectral density function $Q_j(\omega)$. Function $q_j(\tau)$ in the above expression is simply either displacement $U_j(\tau)$ and or acceleration $A_j(\tau)$. Using E_s delivers a statistical estimation of magnitude that is more robust than employing a single peak of the function.

2.2 Bouc-Wen model for rotational spring

The Bouc-Wen hysteretic model is widely used in the literature for systems that exhibit inelastic behaviour under severe cyclic loads. The attractiveness of this approach is that it employs a first order differential equation in terms of an ‘internal hysteretic’ variable y_j to describe, qualitatively the phenomenological nonlinear hysteretic behaviour. The model reproduces the nonlinear hysteretic behaviour of a variety of soils and it is capable of representing complex patterns such as stiffness and strength degradation with cycling loading. The non-dimensional nonlinear moment/rotation function, of j th building foundation, is described by the following Bouc-Wen nonlinear differential equation;

$$\dot{y} = \frac{1}{\gamma_y} \frac{D_j \dot{\theta}_j - v(E) (\varsigma_j |\dot{\theta}_j| |y_j|^{n_j} \text{sgn}(y_j) + \psi_j \dot{\theta}_j |y_j|^{n_j})}{\eta(E)} \quad (10)$$

In the above expression, $\gamma_y = 10^{-4}$ is the strain at the initiation of nonlinear behaviour in the soil that has been defined by Ishibashi and Zhang [33] and Tatsuoka et al. [34]. $[D_j, \varsigma_j, \psi_j, n_j]$ represent the

dimensionless Bouc-Wen parameters that define the shape of the hysteretic stress-strain loops, B_j is the ratio of linear to nonlinear response, δ_v is the strength degradation parameter and δ_η is the stiffness degradation parameter. $v(E)$ and $\eta(E)$ characterize the degradation shape functions, that are dependent to the dissipated hysteretic energy $E(\tau)$ from initial time $\tau = 0$ to the present time τ . In this research is used the values proposed by Gerolymos and Gazetas [35-36] and Drosos et. al [37], that give a reasonable shape for soil spring and damping stress-strain curves for all examined soil profiles. These values provide a good representation of the complex nonlinear characteristics of the cyclic behaviour of the soil element.

Table 2. Bouc-Wen nonlinear soil model parameters

Soil	γ_y	D_j	B_j	ς_j	ψ_j	δ_v	δ_η	n_j
Sand	10^{-4}	1	0.02	0.5	0.5	0.01	0.01	0.6

3. RESULTS

3.1 Ground motion selection

The structure is analysed considering a horizontal ground motion matched with a specific target response spectra. In this way, we significantly reduce the number of nonlinear time-history analyses performed while approximating the mean system response to a set of ground motion time-series that are compatible with the EC8 elastic spectrum. The original ground motion time series is from the event in Imperial Valley California, USA, in 1979 with a magnitude of $M_w=6.5$ and a peak ground acceleration (PGA) equal to $a_{gr} = 0.37g$. This ground motion was obtained from the Pacific Earthquake Engineering Research (PEER) Center Database [38], recorded on weak soils (shear wave velocity equal to 175 m/s). The target horizontal elastic response spectrum $S_a(T)$ chosen in this study was the response spectra defined in Eurocode 8 [39], considering a design ground acceleration equal to $a_g = 0.6g$, ground type equal to “D” (i.e. deposit of loose to medium cohesionless soil with a shear wave velocity $V_s < 180$ m/s).

The Reweighted Volterra Series Algorithm (RVSA) proposed by Alexander et al. [40] is employed. This spectral matching process is stable and robust because it converges to any reasonable response spectrum for any suitable seed time-series and keeps the non-stationary characteristics of the original record.

3.2 Linear and nonlinear response

We evaluate the difference in the dynamic response between the linear and nonlinear cases considering the dynamic coupling of adjacent buildings. For this, we examine the case when two buildings are very close to each other, aspect ratio $s=1.5$ and height ratio $\varepsilon = 1.5$. Figure 2(a) shows the linear (blue line) and nonlinear (red line) response of the buildings 1 and 2 considering the coupled effect in terms of the displacement. Comparing the responses, we observe that the maximum displacement of the buildings increases when the nonlinear behaviour in the soil is included. Likewise, in Figure 2(b) we can observe that the maximum displacement of the buildings 1 and 2, for uncoupled (SSI) case, increase when the nonlinear behaviour in the soil (red line) is assumed. Figure 2(c) shows the power spectral density for the displacements considering four cases: (i) coupled (linear SSSI), (ii) uncoupled (linear SSI) (iii) coupled (nonlinear SSSI) and (iv) uncoupled (nonlinear SSI) response. Comparing the linear and nonlinear responses we observe that building 1 is significantly affected. Building 1’s response power increases by $\chi_{11} = 323[\%]$, for nonlinear SSSI case, in the presence of the taller building 2. Conversely, its response power only increases by $\chi_{11} = 34.6[\%]$, for the linear SSSI case. In an equivalent way for the building 2 has a reduction in response power $\chi_{22} = -57.7[\%]$ (nonlinear SSSI), than in the linear case $\chi_{22} = -20.6[\%]$ (linear SSSI). Thus, we observe that both adverse/beneficial responses can appear greater in the nonlinear SSSI cases.

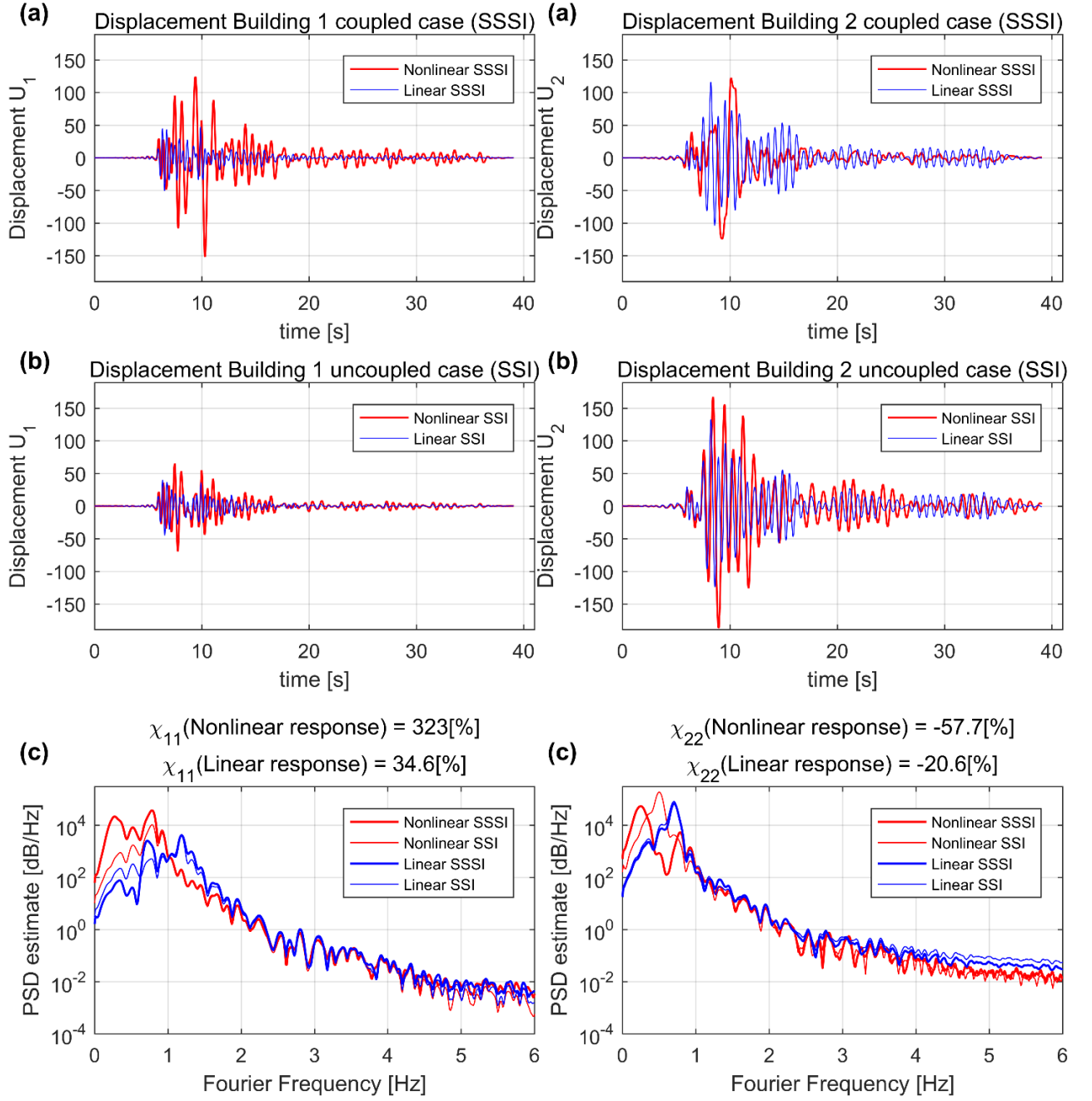


Figure 2. (a) Displacement response coupled case (b) Displacement response uncoupled case (c) power spectra density – Response on loose soil for parameter set ($\epsilon=h_2/h_1=1.5$, $s=h_1/b=1.5$, $z=0.1$).

3.3 Parametric study – variation in aspect and height ratio

Figure 3 displays the contour plots of $\chi_{11}(s, \epsilon)$ for the displacement and acceleration of building 1, for the case with linear behaviour of the soil. The critical zones in the figure are red, i.e. where the buildings 1's total response power is amplified by the presence of building 2 and blue when the response is reduced. The worst possible building parametric configuration lies around $\chi_{11}(0.25, 1.3) = 65\%$ and $\chi_{11}(2.0, 1.2) = 85\%$ for the displacement and acceleration respectively.

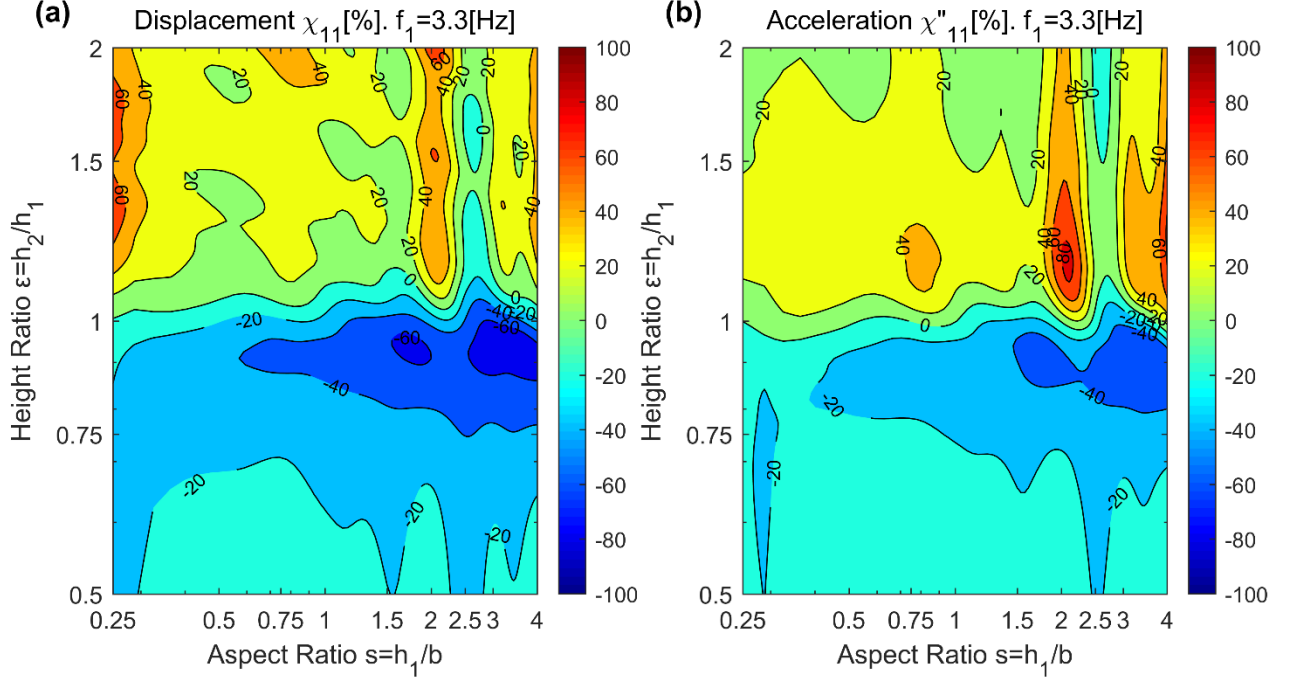


Figure 3. (a) Change in displacement power with the aspect and height ratio (b) Change in acceleration power with the aspect and height ratio – Linear response on loose soil and $z=0.1$.

Contour plots in Figure 4(a) show the variation of change of power for the displacement of the building 1, for the case with nonlinear behaviour of the soil. We consider loose soil and inter-building case equal to $z = 0.1$. In general, it can be observed that the power of earthquake passes from the taller building to the smaller building, increases dramatically when the height ratio is greater than 1.5, reaching values above 400% amplification. Comparing figure 4(a) and figure 3(a) suggests that including nonlinearity flattens the parametric variation in total power responses due to the limiting value of soil-spring capacity assumed in the Bouc-Wen model. Thus, the interaction effect between the buildings increases when the nonlinear behaviour in the soil is considered. This highlights the importance to consider the dynamic coupling (nonlinear SSSI) of critical structures when the structures are very close especially when extreme seismic loads produce predominantly nonlinear behaviour in the system. Figure 4(b) displays the change of power $\dot{\chi}_{11}(s, \varepsilon)$ for the total acceleration of the building 1. In this case, the worst possible configuration is when the second building is 75[%] taller than the first $\dot{\chi}_{11}(0.9, 1.75) = 110$ [%].

Each contour plot, presented in this paper, required approximately 150 hours runtime on the BlueCrystal, the High-Performance Computing (HPC) machine belonging to the Advance computing research centre at the University of Bristol.

Figure 5 repeats the previous analysis for the case of dense sand and a nonlinear analysis case. In this case, the amplification/reduction in the change of power are more limited, $\chi_{11}(2.0, 2.0) = 250$ % and $\dot{\chi}_{11}(3.0, 1.8) = 45$ % to the displacement and acceleration respectively, suggesting that the worst seismic interaction conditions occur on loose soil.

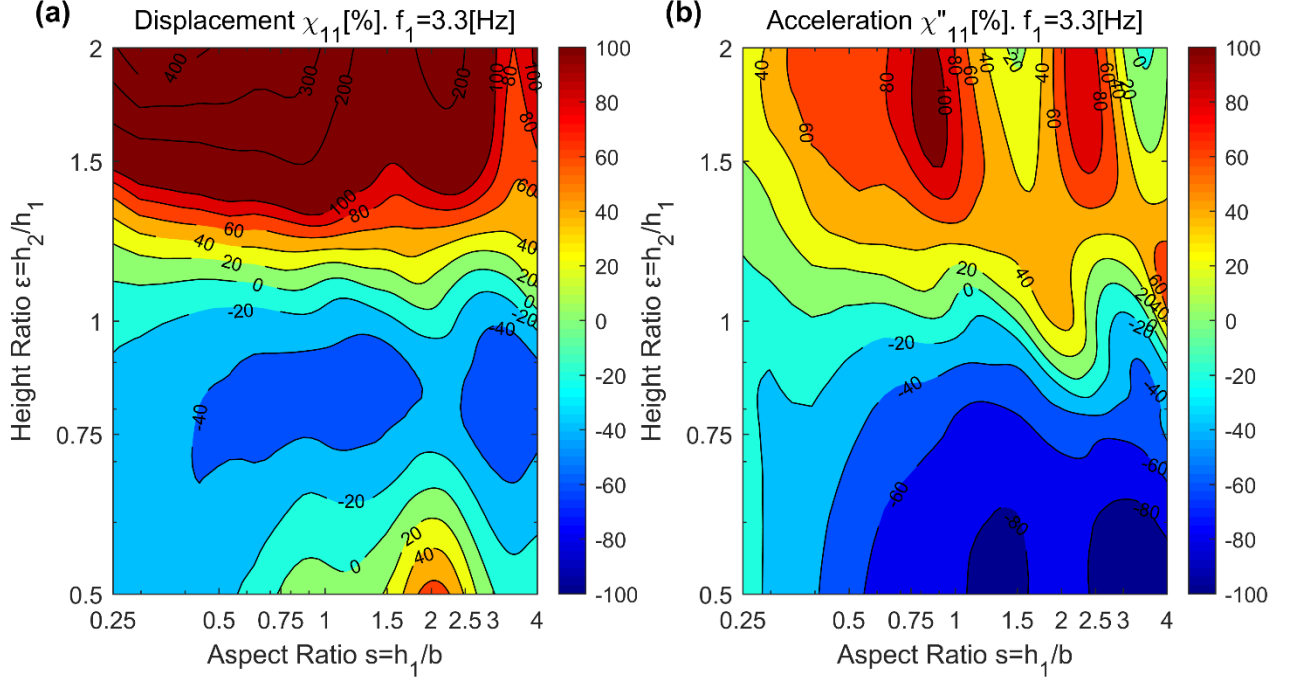


Figure 4. (a) Change in displacement power χ_{11} with the aspect and height ratio (b) Change in acceleration power $\ddot{\chi}_{11}$ with the aspect and height ratio – Nonlinear response in loose soil and $z=0.1$.

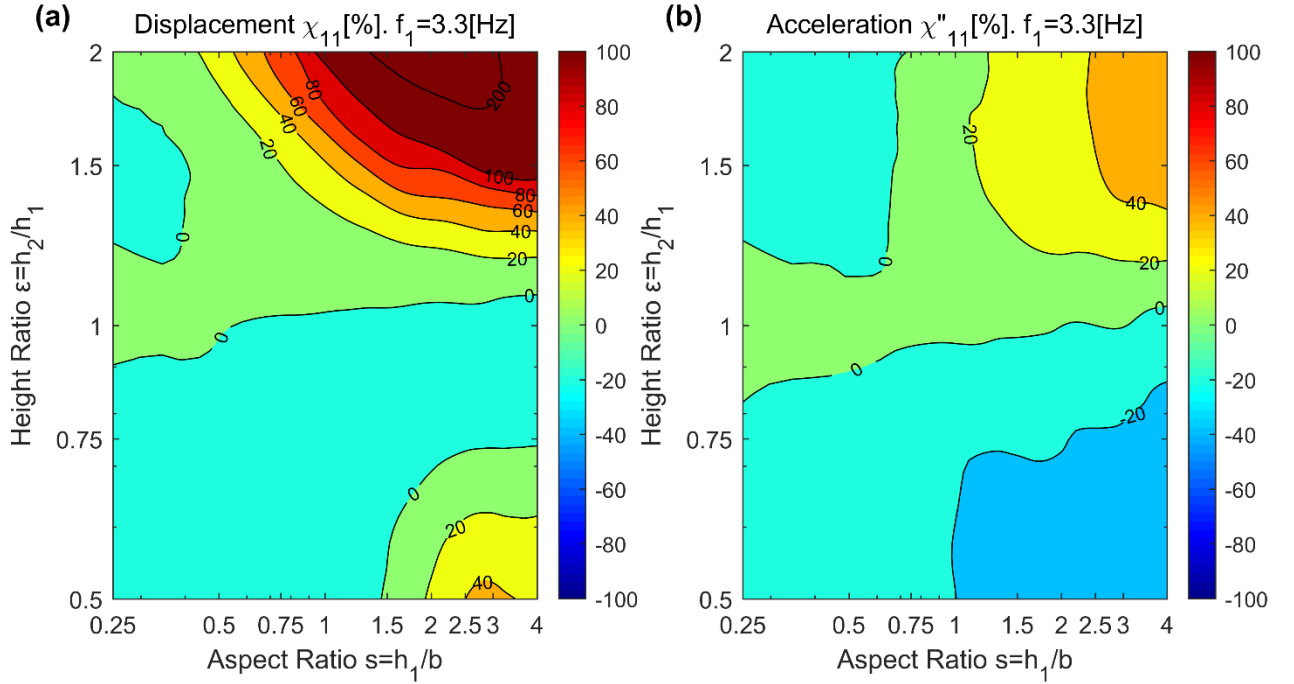


Figure 5. (a) Change in displacement power χ_{11} with the aspect and height ratio (b) Change in acceleration power $\ddot{\chi}_{11}$ with the aspect and height ratio – Nonlinear response on dense soil and $z=0.1$.

3.4 Parametric study – variation in interbuilding spacing

Figure 6(a) shows the variation of power $\chi_{11}(s, \varepsilon, z)$ for the displacement with height ratio $\varepsilon = h_2/h_1$ and inter-building spacing z . The aspect ratio was set equal to $s = 3.0$. As expected the effects of SSSI decreases when increasing the inter-building spacing. At a distance between foundations equal to $2b$, the SSSI is practically negligible $\chi_{11}(3.0, \varepsilon, 2.0) = 4.5\%$. This result happens for any value of aspect

ratio s . Figure 6(b) repeats the previous analysis for the change of power $\ddot{\chi}_{11}(s, \varepsilon, z)$ for the acceleration and similarly, the interaction effect drops more sharply with increasing the inter-building spacing to a value of $\ddot{\chi}_{11}(3.0, \varepsilon, 2.0) = 3.8\%$.

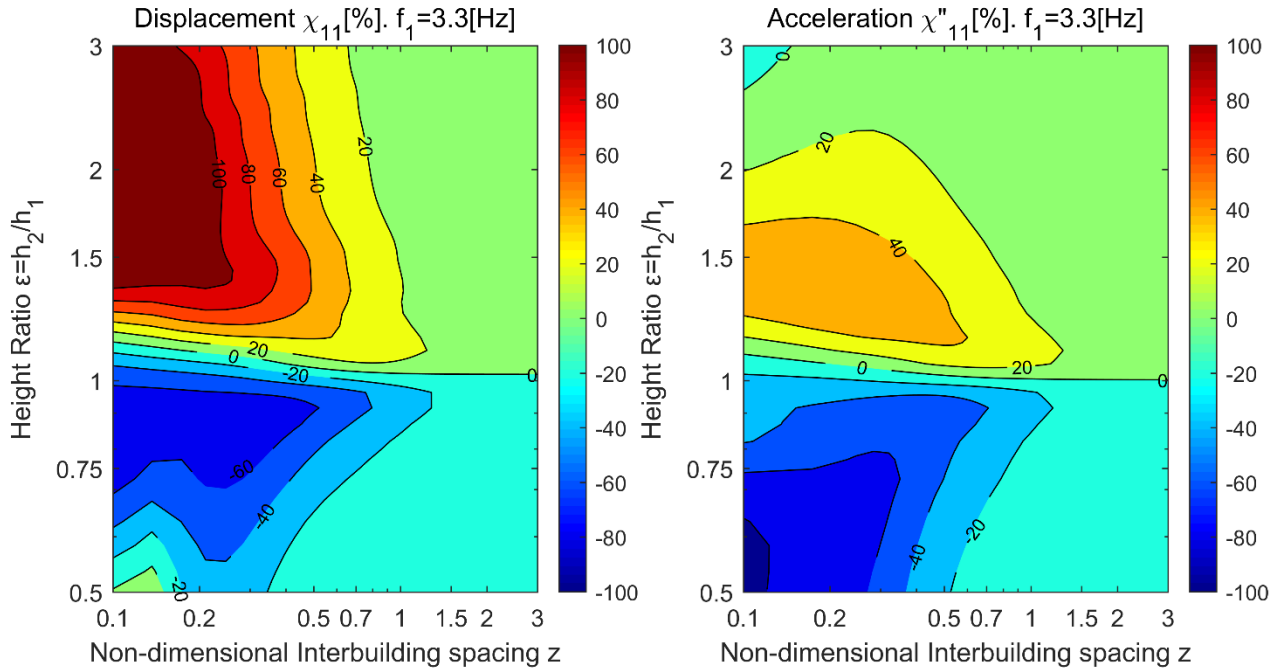


Figure 6. (a) Change in displacement power χ_{11} with the height ratio and inter-building spacing (b) Change in acceleration power with the height ratio and inter-building spacing – Nonlinear response on loose soil and $s=3.0$.

4. CONCLUSIONS

A 2-D Structure-Soil-Structure Interaction formulation between two buildings is proposed. The buildings are coupled through the soil and it is considered a nonlinear phenomenological Bouc-Wen model for the soil underneath the foundations. The seismic ground motion employed is spectrally matched with EC8 elastic spectra.

The nonlinear SSSI parametric study showed that there are significant differences in comparison with the response to the linear SSSI analysis. It is found that the nonlinear SSSI can produce a greater range of beneficial and adverse behaviour for displacement than linear SSSI, which highlights the importance of considering nonlinear SSSI analysis in critical structures (such as nuclear reactor structures, hospitals and towers). There are significant differences between the nonlinear SSSI (coupling building case) and nonlinear SSI (uncoupled building case). The most adverse effects, on building displacement, occurred when there is a big difference of height ($\varepsilon > 1.5$) between the buildings. In this case, the displacement power of building 1 can be amplified to 400%, i.e. the power of the earthquake passed from the taller structure to the small structure. The most adverse effects, on building acceleration, can be as high as 110% and occurred when a smaller building 1 is flanked by a taller building 2.

Results from well-spaced building, around 2 times the building base width, show that the SSSI seismic response energy amplification is negligible. For dense soil, the results show that the SSSI interaction is less relevant than for the case of loose soil.

Therefore, this research indicates that to assess the risk in important structures is necessary to consider the nonlinear SSSI effects, so the interaction between structures is an effect that should not be neglected.

6. ACKNOWLEDGMENTS

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